AD-A195 392 REPAIR EVALUATION MAINTENANCE AND REHABILITATION RESEARCH PROGRAM: STABIL (U) COASTAL ENGINEERING RESEARCH CENTER VICKSBURG MS R D CARVER ET AL JUN 88

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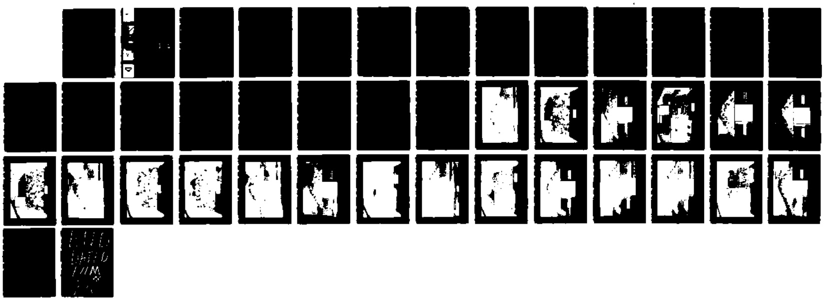

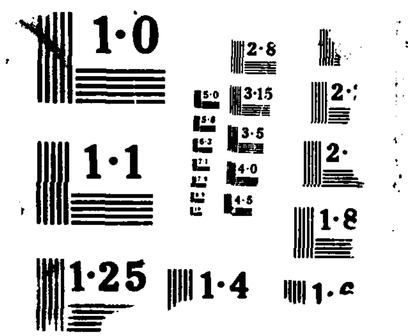
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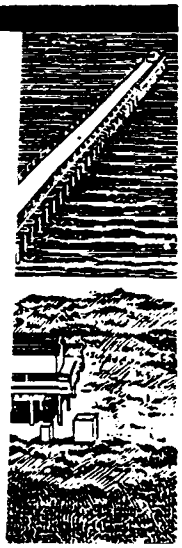

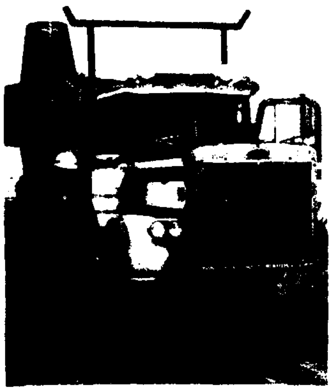

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REPAIR, EVALUATION, MAINTENANCE, AND REHABILITATION RESEARCH PROGRAM

TECHNICAL REPORT REMR-CO-5

STABILITY OF DOLOS OVERLAYS FOR REHABILITATION OF DOLOS-ARMORED RUBBLE-MOUND BREAKWATER AND JETTY TRUNKS SUBJECTED TO BREAKING WAVES

by

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DEPARTMENT OF THE ARMY
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June 1988

Final Report

Approved For Public Release; Distribution Unlimited

Prepared for DEPARTMENT OF THE ARMY
US Army Corps of Engineers
Washington, DC 20314-1000

Under REMR Work Unit 32325

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The following two letters used as part of the number designating technical reports of research published under the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program identify the problem area under which the report was prepared:

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Unclassified
SECURITY CLASSIFICATION OF THIS PAGE

REPORT DOCUMENTATION PAGE

Form Approved
OMB No 0704-0188
Exp Date Jun 30 1986

1a REPORT SECURITY CLASSIFICATION Unclassified	1b RESTRICTIVE MARKINGS
2a SECURITY CLASSIFICATION AUTHORITY	3 DISTRIBUTION / AVAILABILITY OF REPORT Approved for public release; distribution unlimited.
2b DECLASSIFICATION / DOWNGRADING SCHEDULE	
4 PERFORMING ORGANIZATION REPORT NUMBER(S) Technical Report REMR-CO-5	5 MONITORING ORGANIZATION REPORT NUMBER(S)
6a NAME OF PERFORMING ORGANIZATION USAEWES, Coastal Engineering Research Center 6b OFFICE SYMBOL (If applicable)	7a NAME OF MONITORING ORGANIZATION
6c ADDRESS (City, State, and ZIP Code) PO Box 631 Vicksburg, MS 39180-0631	7b ADDRESS (City, State, and ZIP Code)
8a NAME OF FUNDING / SPONSORING ORGANIZATION US Army Corps of Engineers 8b OFFICE SYMBOL (If applicable)	9 PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER
8c ADDRESS (City, State, and ZIP Code) Washington, DC 20314-1000	10 SOURCE OF FUNDING NUMBERS

PROGRAM ELEMENT NO	PROJECT NO	TASK NO	WORK UNIT ACCESSION NO
			32325

11 TITLE (Include Security Classification)
Stability of Dolos Overlays for Rehabilitation of Dolos-Armored Rubble-Mound Breakwater and Jetty Trunks Subjected to Breaking Waves

12 PERSONAL AUTHOR(S)
Carver, Robert D.; Wright, Brenda J.

13a TYPE OF REPORT	13b TIME COVERED	14 DATE OF REPORT (Year, Month, Day)	15 PAGE COUNT
Final report	FROM ______ TO ______	June 1988	42

16 SUPPLEMENTARY NOTATION
Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

17 COSATI CODES		
FIELD	GROUP	SUB-GROUP

18 SUBJECT TERMS (Continue on reverse if necessary and identify by block number)
Armor units; Jetties; Breakwaters; Rubble mound;

19 ABSTRACT (Continue on reverse if necessary and identify by block number)

An experimental model investigation was conducted to obtain design guidance for dolos overlays used to rehabilitate dolos-armored rubble-mound breakwater and jetty trunks subjected to breaking waves. All tests were conducted with a 1V-on-10H slope fronting the test sections and 1V- on-1.5H- and 1V- on-2H-structure slopes. It was concluded that:

a. Stability showed some dependency on both d/L and H/d, with minimum stability occurring at the lower values of d/L and higher values of H/d, i.e. longer wave periods in shallower water.

b. The minimum stability coefficient observed is very similar to that presently recommended for new construction (15.6 versus 15).

20 DISTRIBUTION / AVAILABILITY OF ABSTRACT ☒ UNCLASSIFIED/UNLIMITED ☐ SAME AS RPT ☐ DTIC USERS	21 ABSTRACT SECURITY CLASSIFICATION Unclassified
22a NAME OF RESPONSIBLE INDIVIDUAL	22b TELEPHONE (Include Area Code) / 22c OFFICE SYMBOL

DD FORM 1473, 84 MAR — 83 APR edition may be used until exhausted. All other editions are obsolete

SECURITY CLASSIFICATION OF THIS PAGE
Unclassified

PREFACE

Authority to carry out this investigation was granted the US Army Engineer Waterways Experiment Station's (CEWES's) Coastal Engineering Research Center (CERC) by the Office, Chief of Engineers (OCE), under the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program Work Unit 32325, "Use of Dissimilar Armor for Repair and Rehabilitation of Rubble-Mound Coastal Structures."

Tests of dolos overlays for existing dolos armor, which fulfill one milestone of this work unit, were conducted under the general direction of Messrs. James E. Crews and Tony C. Liu, REMR Overview Committee, OCE; Jesse A. Pfeiffer, Jr., Directorate of Research and Development, OCE; John H. Lockhart, Jr., REMR Coastal Problem Area Monitor, OCE; William F. McCleese, REMR Program Manager, CEWES; and D. D. Davidson, REMR Coastal Problem Area Leader, CERC.

The study was conducted by personnel of CERC under general direction of Dr. James R. Houston, Chief, CERC, and Mr. Charles C. Calhoun, Jr., Assistant Chief, CERC; and under direct supervision of Messrs. C. E. Chatham, Chief, Wave Dynamics Division, and D. D. Davidson, Wave Research Branch. Tests were planned by Mr. Robert D. Carver, Principal Investigator, and Ms. Brenda J. Wright, Civil Engineering Technician. The model was operated by Ms. Wright under supervision of Mr. Carver, and this report was prepared by Mr. Carver and Ms. Wright. This report was edited by Ms. Shirley A. J. Hanshaw, Information Products Division, Information Technology Laboratory, CEWES.

Commander and Director of CEWES at the time of report publication was COL Dwayne G. Lee, CE. Technical Director was Dr. Robert W. Whalin.

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CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
feet	0.3048	metres
inches	25.4	millimetres
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square feet	0.09290304	square metres

STABILITY OF DOLOS OVERLAYS FOR REHABILITATION OF DOLOS-ARMORED RUBBLE-MOUND BREAKWATER AND JETTY TRUNKS SUBJECTED TO BREAKING WAVES

PART I: INTRODUCTION

Background

1. The experimental investigation described herein constitutes a portion of a research effort to provide engineering data for the effective and economical rehabilitation of rubble-mound breakwaters and jetties. In this study, a rubble-mound breakwater or jetty is defined as a protective structure constructed with a core of quarry-run stone, sand, or slag and protected from wave action by one or more stone underlayers and a cover layer composed of selected quarrystone or specially shaped concrete armor units.

2. Previous investigations under Work Unit 31269, "Stability of Breakwaters," have yielded significant design information for new construction using quarrystone (Hudson 1958 and Carver 1980, 1983), tetrapods, quadripods, tribars, modified cubes, hexapods, and modified tetrahedrons (Jackson 1968), dolosse (Carver and Davidson 1977 and Carver 1983), and toskane (Carver 1978). Rehabilitation projects on several of the Corps' rubble-mound structures have revealed a total lack of design guidance or information concerning the interfacing and stability response of armor units that are of dissimilar type and/or size. In the past, selection of new armor type, method of interfacing, and procedures for preparation of the existing section have been based on engineering judgment or, in more recent times, on site-specific model studies. The engineering judgment process may be expensive since experience is limited and there is not usually a solid basis for it. This process can lead to recurring failures that cost millions of dollars without a real solution being developed for the long-term problem. Site-specific model studies have provided good singular solutions, but site-specific data usually fail to meet the requirements of other projects (Carver, in preparation). It is anticipated that the problem will become more acute in future years as rehabilitation of major breakwaters and jetties becomes necessary to extend their project life or to meet greater design demands.

Approach

3. Model breakwaters and armor units are being used to experimentally investigate the stability response of various armor combinations for selected structure geometries and wave conditions. It would be an extremely extensive task to comprehensively investigate all different types of existing armor units; therefore, this research effort will address only the three types (stone, dolos, and tribars) of armor most commonly used in the Corps. Selection of these armor types should give test results the widest range of applicability possible. Tests were conducted with breaking wave conditions on no-damage, no-overtopping breakwater trunk and head sections using sea-side slopes of 1V on 1.5H and 1V on 2H. Test results for dolos and tribar overlays of existing stone armor have been reported (Carver and Wright, 1988).

Purpose of Study

4. The purpose of the present investigation was to obtain design guidance for dolos overlays used to rehabilitate dolos-armored rubble-mound breakwater and jetty trunks subjected to breaking waves. More specifically, it was desired to determine the minimum weight of individual armor units (with given specific weights) required for stability as a function of:

- a. Sea-side slope of the structure.
- b. Wave period.
- c. Wave height.
- d. Water depth.

PART II: TESTS

Stability Scale Effects

5. If the absolute sizes of experimental breakwater materials and wave dimensions become too small, flow around the armor units enters the laminar regime; and the induced drag forces become a direct function of the Reynolds number. Under these circumstances prototype phenomena are not properly simulated, and stability scale effects are induced. Hudson (1975) presents a detailed discussion of the design requirements necessary to ensure the preclusion of stability scale effects in small-scale breakwater tests and concludes that scale effects will be negligible if the Reynolds stability number R_N *

$$R_N = \frac{g^{1/2} H^{1/2} \ell_a}{\nu}$$

where

g = acceleration due to gravity, ft/sec^2

H = wave height, ft

ℓ_a = characteristic length of armor unit, ft

ν = kinematic viscosity

is equal to or greater than 3×10^4. For all tests reported herein, the sizes of experimental armor and wave dimensions were selected such that scale effects were insignificant (i.e., R_N was greater than 3×10^4).

Test Procedures

Method of constructing test sections

6. All experimental breakwater sections were constructed to reproduce as closely as possible results of the usual methods of constructing full-scale breakwaters. The core material was dampened as it was dumped by bucket or shovel into the flume and was compacted with hand trowels to simulate natural consolidation resulting from wave action during construction of the prototype

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* For convenience, symbols and unusual abbreviations are listed and defined in the Notation (Appendix A).

structure. Once the core material was in place, it was sprayed with a low-velocity water hose to ensure adequate compaction of the material. The underlayer stone then was added by shovel and smoothed to grade by hand or with trowels. No excessive pressure or compaction was applied during placement of the underlayer stone. Armor units used in the cover layers were placed in a random manner corresponding to work performed by a general coastal contractor, i.e., they were individually placed but were laid down without special orientation or fitting. After each test series the armor units were removed from the breakwater, all of the underlayer stones were replaced to the grade of the original test section, and the armor was replaced.

Selection of critically breaking waves

7. For a given wave period and water depth, the most detrimental breaking wave (i.e. the most damaging wave) was determined by increasing the stroke adjustment on the wave generator in small increments and observing which wave produced the most severe breaking wave condition on the experimental structures. Wave heights of lower amplitude did not form the critical breaking wave, and wave heights of larger amplitude would break seaward of the test structures and dissipate their energy so that they were less damaging than the critically tuned wave.

8. A typical stability test series consisted of subjecting the test sections to attack by waves of given heights and periods until all damage had abated or the structures failed. Test sections were subjected to wave attack in approximately 30-sec intervals between which the wave generator was stopped and the waves were allowed to decay to zero height. This procedure was necessary to prevent the structures from being subjected to an undefined wave system created by reflections from the experimental breakwater and wave generator. Newly built test sections were subjected to a short duration (five or six 30-sec intervals) of shakedown using a wave equal in height to about one-half of the design wave. This procedure provided a means of allowing consolidation and armor unit seating that would normally occur during prototype construction.

Method of determining damage

9. To evaluate and compare breakwater stability test results, it is necessary to quantify the changes that have taken place in a given structure during attack by waves of specified characteristics. During the early 1950's, the US Army Engineer Waterways Experiment Station (CEWES) developed a method

of measuring the percent damage incurred by a test section. This method has proven satisfactory and was used as a means for analyzing and comparing the stability tests delineated herein.

10. The CEWES damage-measurement technique requires that the cross-sectional area occupied by armor units be determined for each stability test section. Armor unit area is computed from elevations (soundings) taken at closely spaced grid-point locations before the armor is placed on the underlayer, after the armor has been placed but before the section has been subjected to wave attack, and finally after wave attack. Elevations are obtained with a sounding rod equipped with a circular spirit level for plumbing, a scale graduated in thousandths of a foot, and a ball-and-socket foot for adjustment to the irregular surface of the breakwater slope. The diameter (in inches) of the circular foot of the sounding rod was related to the size of the material being sounded by the following equation:

$$\text{Diam} = C\left(\frac{W_a}{\gamma_a}\right)^{1/3}$$

where

C = 13.7 for dolosse

W_a = weight of an armor unit, lb

γ_a = specific weight of armor unit, pcf

A series of sounding tests in which both the weight of the armor and the diameter of the sounding foot were varied indicated that the above relation would give a measured thickness which visually appeared to represent an acceptable two-layer thickness.

11. Sounding data for each test section were obtained in the following manner. After the underlayer was in place, soundings were taken on the slopes of the structure along rows beginning at and parallel to the longitudinal center line of the structure and extending in 0.25-ft* horizontal increments until the edge of the armor was reached. On each parallel row, sounding points, spaced at 0.25-ft increments, were measured. The 0.5 ft of structure

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

next to each wall was not considered because of the possibility of discontinuity effects between armor units and the flume walls. Soundings were taken at the same points once the armor was in place and again after the structure had been subjected to wave attack.

12. Sounding data from each stability test were reduced in the following manner. The individual sounding points obtained on each parallel row were averaged to yield an average elevation at the bottom of the armor layer before the armor was placed and then at the top of the armor layer before and after testing. From these values the cross-sectional armor area before testing and the area from which armor units were displaced (either downslope or off the section) were calculated. Damage was then determined from the following relation:

$$\text{Percent damage} = \frac{A_2}{A_1} (100)$$

where

A_1 = area before testing, ft^2

A_2 = area from which armor units have been displaced, ft^2

The percentage given by the CEWES sounding technique is, therefore, a measurement of an end area which converts to an average volume of armor material that has been moved from its original location (either downslope or off-structure).

Test Equipment

13. All tests were conducted in a 5-ft-wide, 4-ft-deep, 119-ft-long concrete wave flume with test sections installed about 90 ft from a vertical displacement wave generator. A thin divider was installed in the center of the test section area, thus yielding two 2.5-ft-wide sections. The first 10-ft length of flume bottom, immediately seaward of the test sections, was molded on a 1V-on-10H slope, while the remaining 80-ft length was flat. The generator is capable of producing sinusoidal waves of various periods and heights. For all tests, waves of the required characteristics were generated by varying the frequency and amplitude of the plunger motion. Changes in water surface elevation as a function of time (wave heights) were measured by

electrical wave height gages in the vicinity of where the toe of the test sections was to be placed (without the structure in place) and recorded on chart paper by an electrically operated oscillograph. The electrical output of the wave gages was directly proportional to their submergence depth.

Selection of Test Conditions

14. Breaking wave tests were conducted using dolos overlays. A review of past site-specific stability projects and hydrographic data showed that typical prototype sea-bottom slopes could range from almost flat to as steep as 1V on 10H. Realizing that wave deformation and severity of breaking action increases as bottom slope increases and since time constraints would allow testing of only one foreslope, it was decided to use a 1V-on-10H slope, thus ensuring severe depth-limited breaking wave action (plunging breakers). When breaking directly on the structure, this type of wave normally causes the most damage to rubble-mound structures.

15. By nondimensionalizing design conditions from site-specific projects, it was found that a relative depth d/L range of 0.04 to 0.14 should include most prototype conditions encountered in breaking wave stability designs. A review of capabilities of the available flume and wave generator showed that this range of d/L values could be achieved for a reasonable range of testing depths.

16. The wave flume was calibrated for depths from 0.40 to 0.95 ft in 0.05-ft increments at d/L values of 0.04, 0.06, 0.08, 0.10, 0.12, and 0.14. This range of depths, and consequently breaking wave heights, proved to be compatible with the selected armor weights and sea-side breakwater slopes.

17. All stability tests were conducted on sections of the type shown in Figure 1 and Photos 1-4. Sea-side slopes of 1V on 1.5H and 1V on 2H were investigated, while the beach-side slope was held constant at 1V on 1.5H. Heights of the simulated existing structures (prior to placement of the dolos overlays) varied from 1.0 to 1.2 ft. The height necessary to prevent wave overtopping of the existing structure was determined from slopes and estimated water depths and wave heights to be investigated in determining stability coefficients for the dissimilar armor overlays.

18. It was assumed that the overlaying dolos armor would need to be slightly to significantly larger than the existing dolosse to achieve

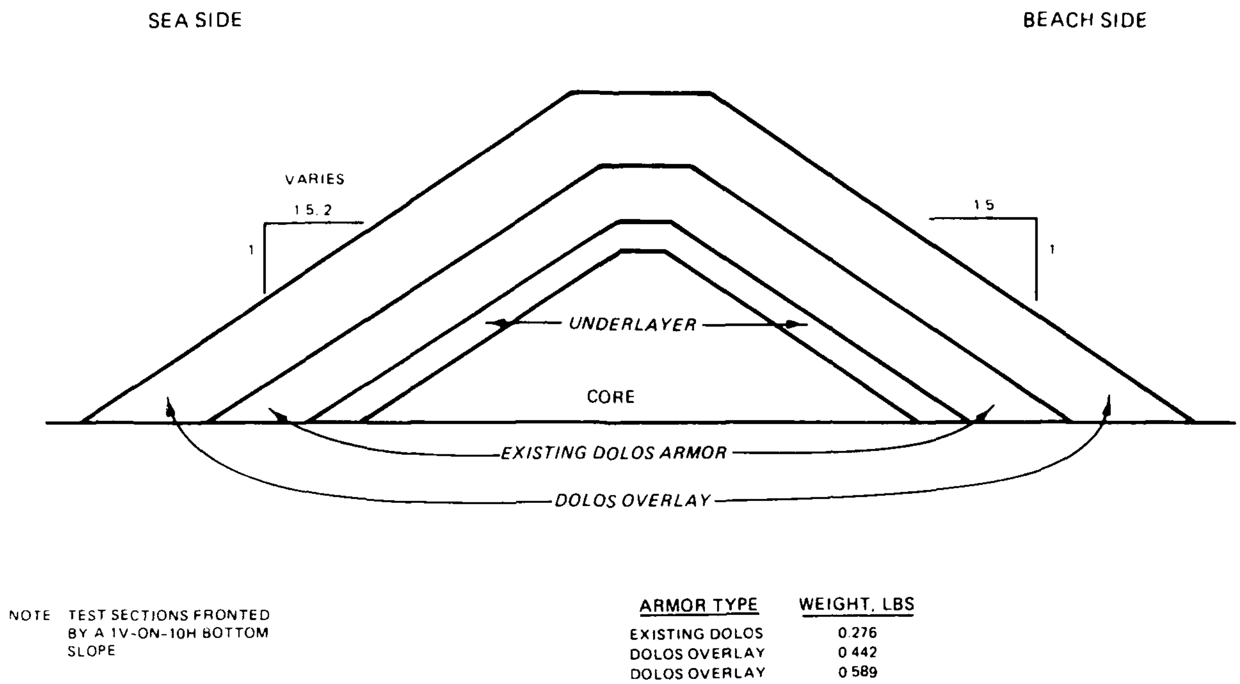

Figure 1. Typical breakwater cross section

stability. A review of existing model materials was made in concert with this assumption, and 0.276-lb dolosse were selected to simulate existing conditions. Tests were conducted with 0.442- and 0.589-lb overlays.

PART III: TEST RESULTS

19. Stability test results are summarized in Table 1. Presented therein are experimentally determined stability coefficients K_D's as functions of relative depth d/L and relative wave height H/d . The stability coefficient K_D is determined from the Hudson formula, i.e.,

$$W_a = \frac{\gamma_a H^3}{K_D \left(S_a - 1\right)^3 \cot \alpha}$$

where

K_D = stability coefficient

S_a = specific gravity of armor unit

α = reciprocal of breakwater slope

Armor units were placed randomly in two layers, and the number of armor units per given surface area was equal to that presently recommended for new construction in EM 1110-2-2904 (Headquarters, Department of the Army 1986). Photos 5-20 show typical after-testing conditions of the structures.

20. Figures 2 and 3 present K_D as a function of d/L and H/d , respectively. These data show some dependency on both d/L and H/d with minimum stability occurring at the lower values of d/L and higher values of H/d , i.e. longer wave periods in shallower water. These trends are consistent with those observed by Carver (1983) for dolos used in new construction. The minimum stability coefficient observed is very similar to that currently recommended for new construction (15.6 versus 15).

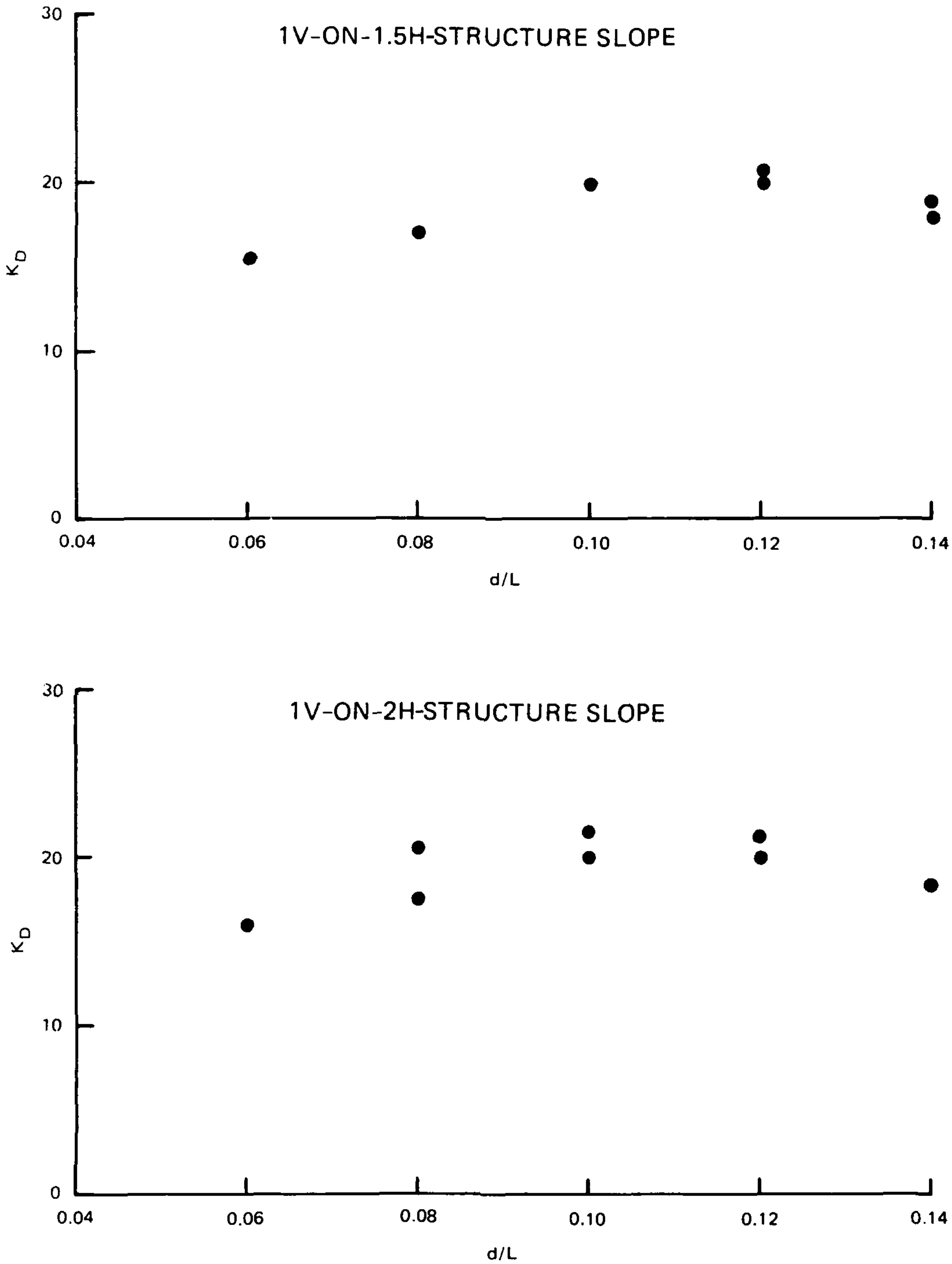

Figure 2. Stability coefficient K_D versus relative depth d/L

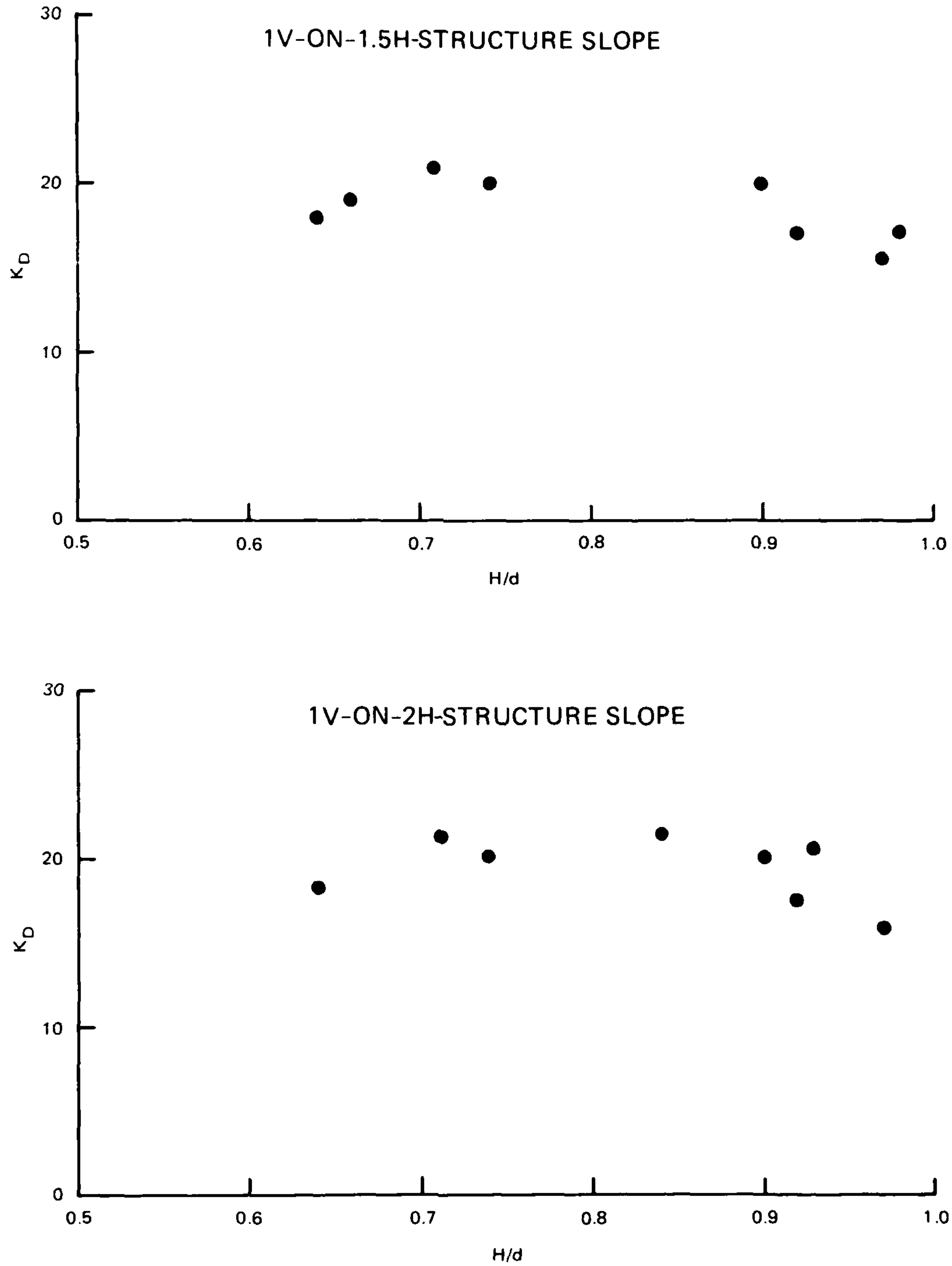

Figure 3. Stability coefficient K_D versus relative wave height H/d

PART IV: CONCLUSIONS

21. Based on tests and results described herein in which dolos armor is used to overlay existing dolos on breakwater trunks subjected to breaking waves with a direction of approach of 90 deg, it is concluded that:

a. Stability showed some dependency on both d/L and H/d, with minimum stability occurring at the lower values of d/L and higher values of H/d, i.e., longer wave periods in shallower water.

b. The minimum stability coefficient observed is very similar to that presently recommended for new construction (15.6 versus 15).

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Table 1

Values of H, d/L, H/d, and K_D for Dolos Overlays of Existing Dolos Armor Subjected to Breaking Waves

W_a, lb	d, ft	T, sec	H, ft	d/L	H/d	K_D
			1V-on-1.5H-Structure Slope			
0.442	0.55	1.70	0.54	0.08	0.98	17.0
0.442	0.85	1.30	0.56	0.14	0.66	18.9
0.589	0.60	2.32	0.58	0.06	0.97	15.6
0.589	0.65	1.85	0.60	0.08	0.92	17.1
0.589	0.70	1.57	0.63	0.10	0.90	19.9
0.589	0.85	1.47	0.63	0.12	0.74	19.9
0.589	0.90	1.52	0.64	0.12	0.71	20.8
0.589	0.95	1.37	0.61	0.14	0.64	18.0
			1V-on-2H-Structure Slope			
0.442	0.60	2.32	0.58	0.06	0.97	15.8
0.442	0.65	1.85	0.60	0.08	0.92	17.5
0.442	0.70	1.57	0.63	0.10	0.90	20.2
0.442	0.85	1.47	0.63	0.12	0.74	20.2
0.442	0.90	1.52	0.64	0.12	0.71	21.3
0.442	0.95	1.37	0.61	0.14	0.64	18.3
0.589	0.75	1.99	0.70	0.08	0.93	20.5
0.589	0.85	1.73	0.71	0.10	0.84	21.4

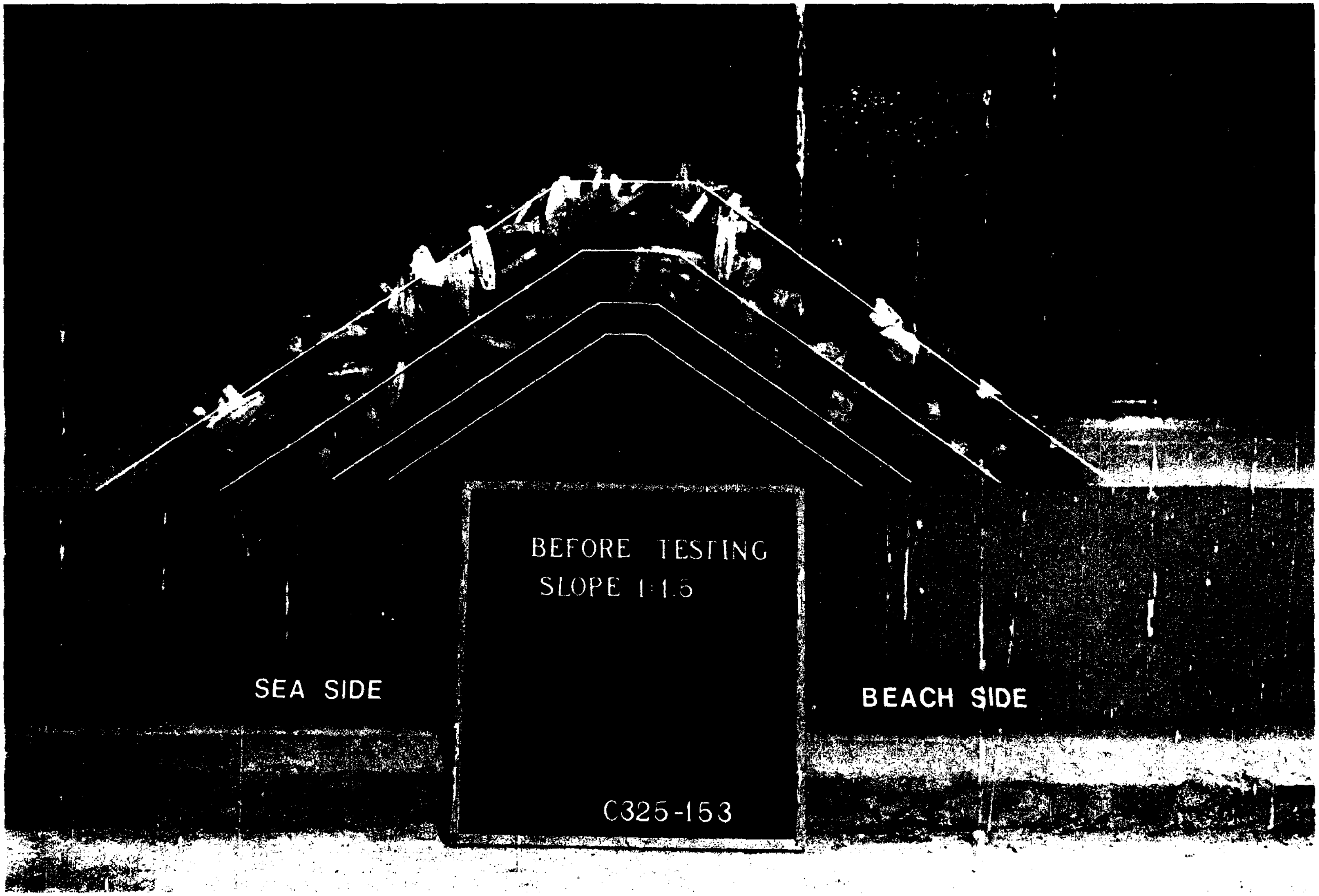

Photo 1. End view of a typical test section before wave attack at a 1V-on-1.5H-sea-side-structure slope; $W_a = 0.589$ lb

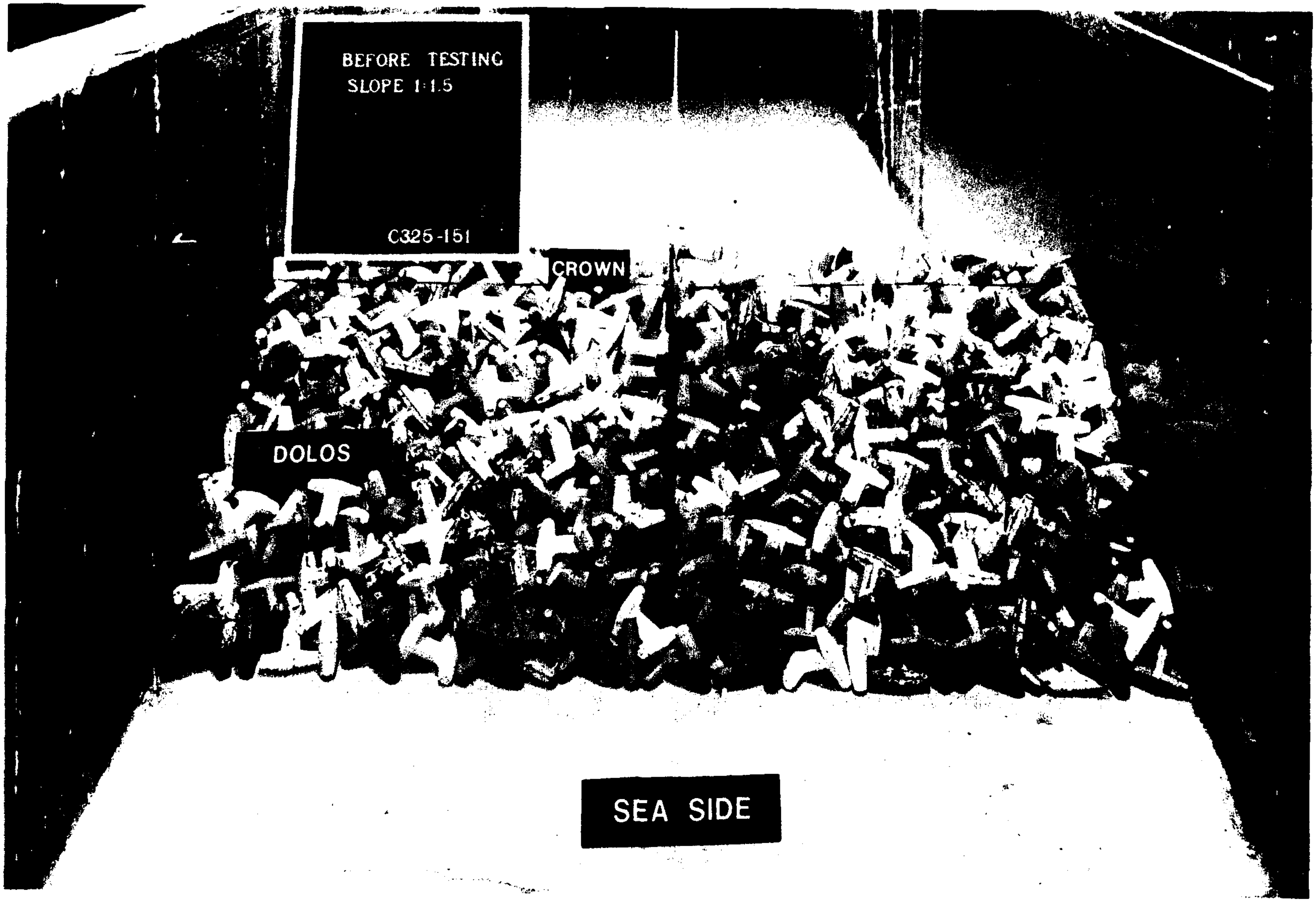

Photo 2. Sea-side view of a typical test section before wave attack at a 1V-on-1.5H-sea-side-structure slope; W_a = 0.589 lb

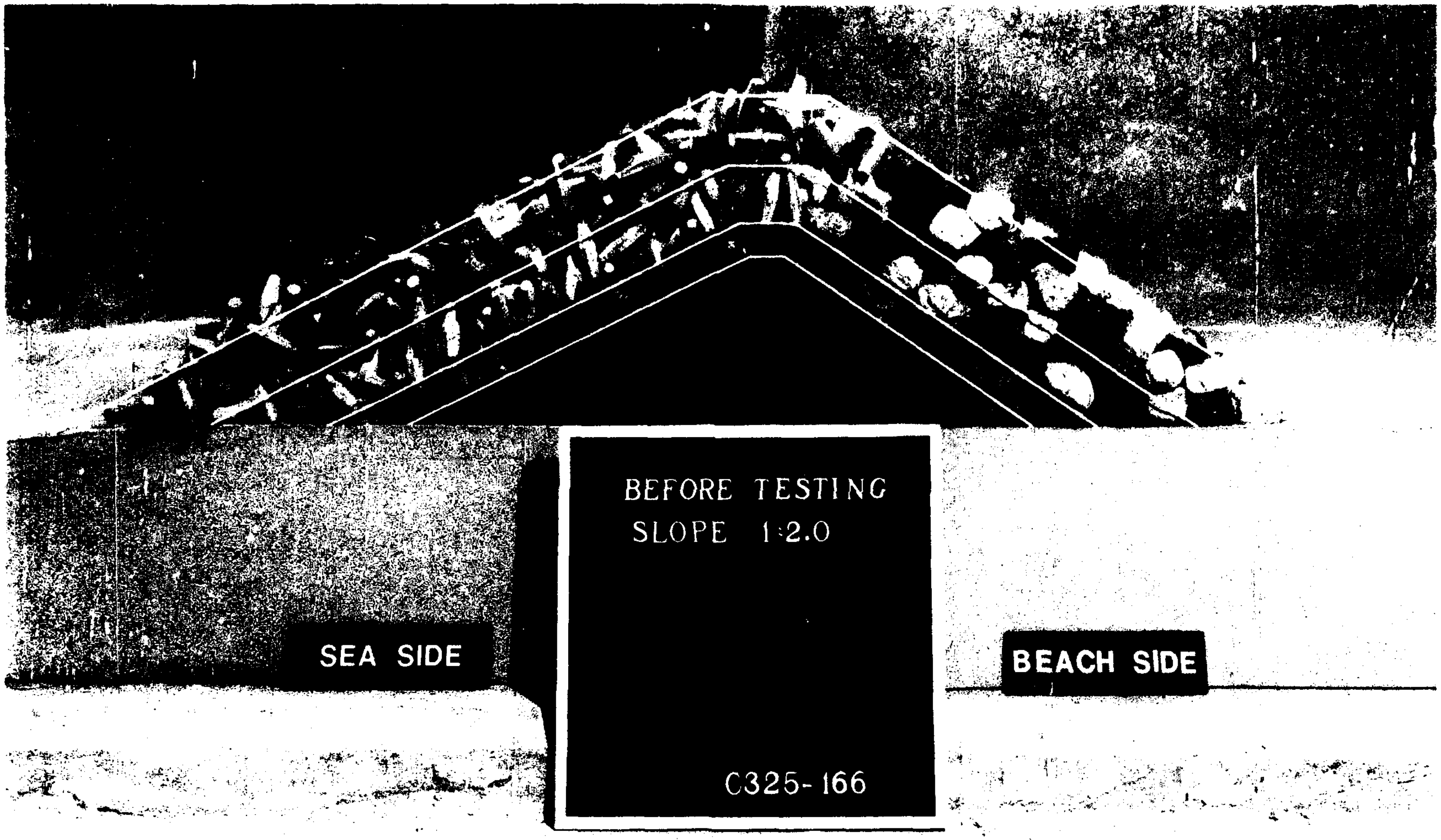

Photo 3. End view of a typical test section before wave attack at a 1V-on-2H-sea-side-structure slope; W_a = 0.442 lb

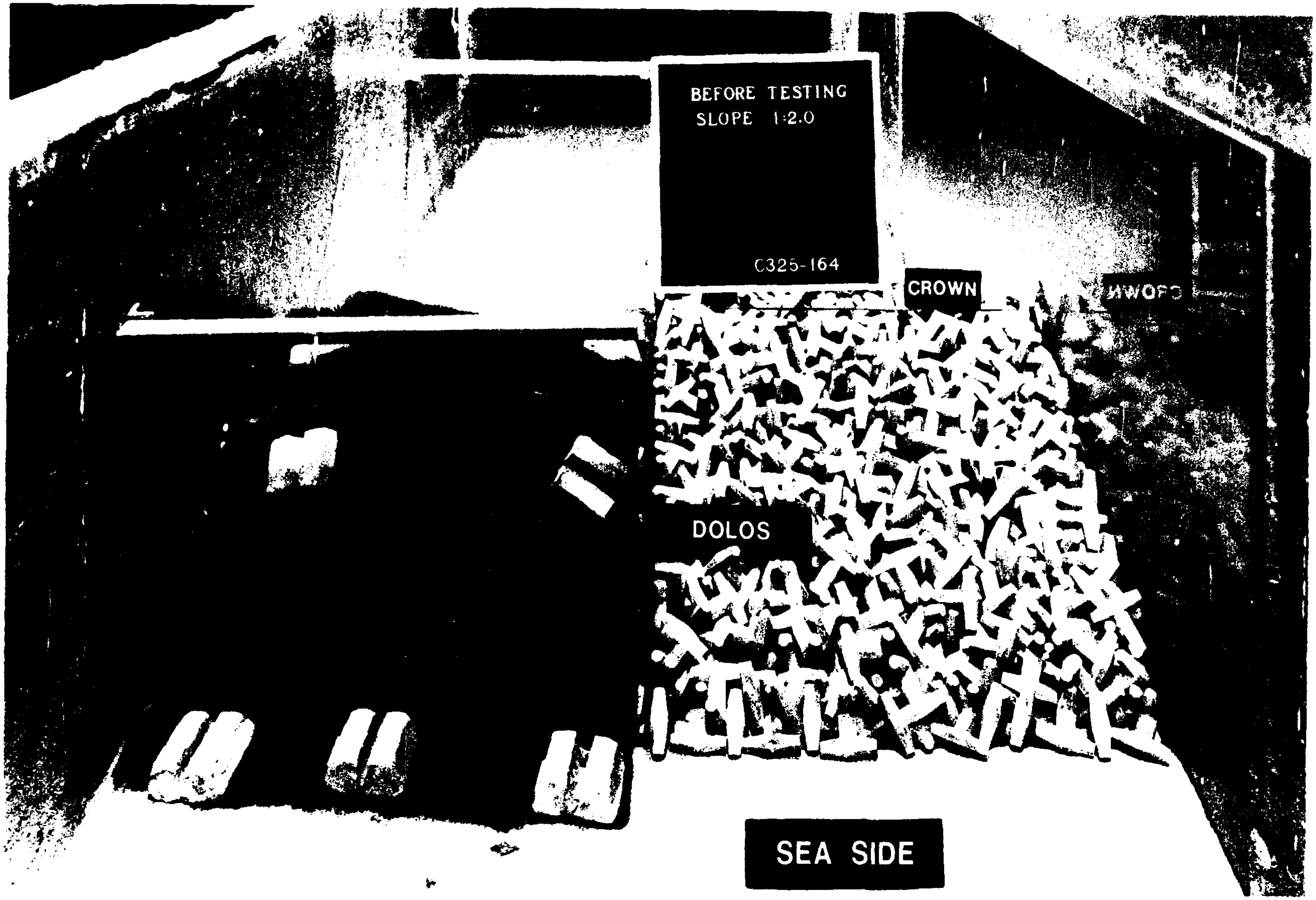

Photo 4. Sea-side view of a typical test section before wave attack at a 1V-on-2H-sea-side-structure slope; W_a = 0.442 lb

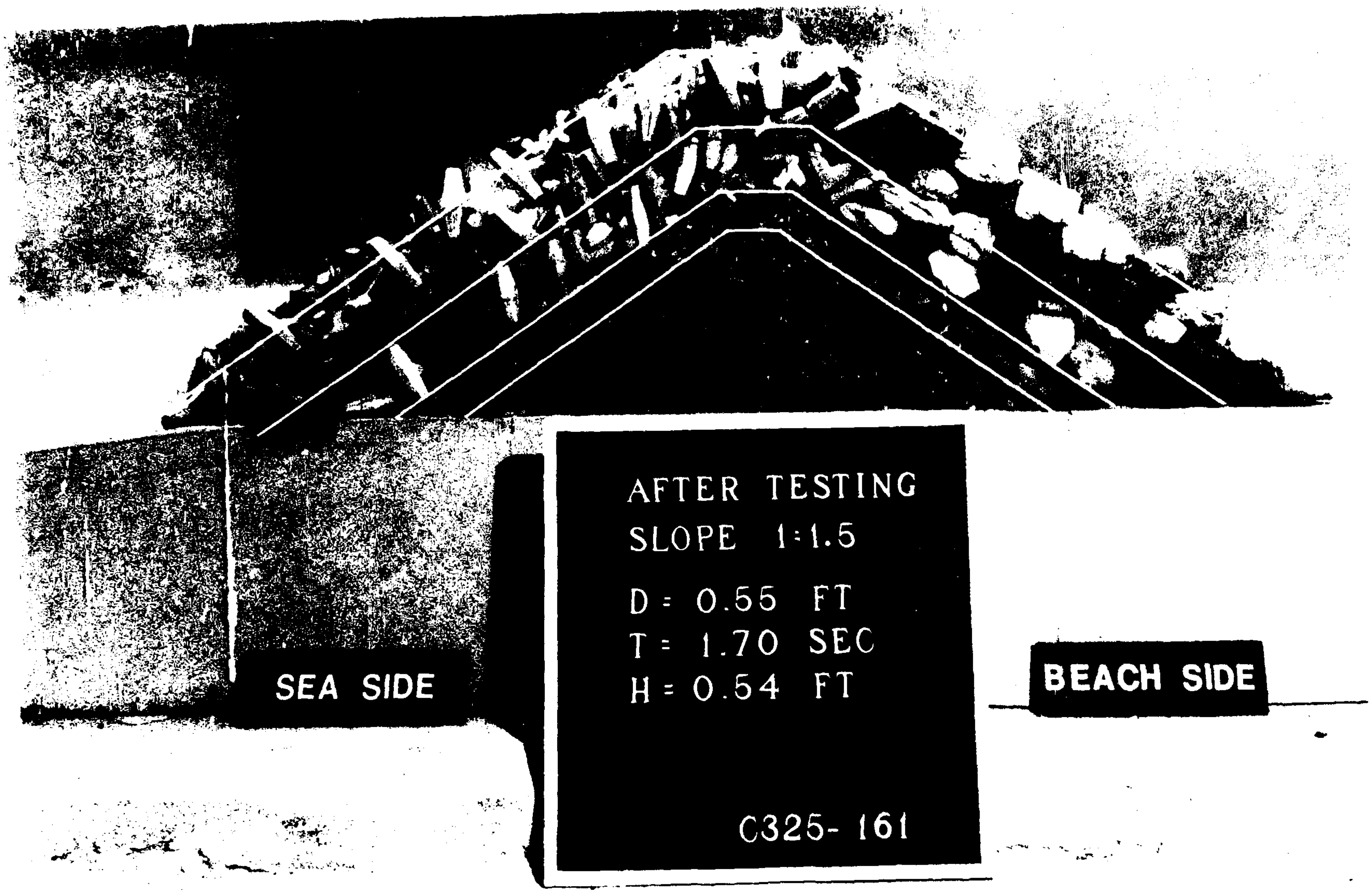

Photo 5. End view after attack of 1.70-sec, 0.54-ft waves; d = 0.55 ft; W_a = 0.442 lb; 1V-on-1.5H-structure slope

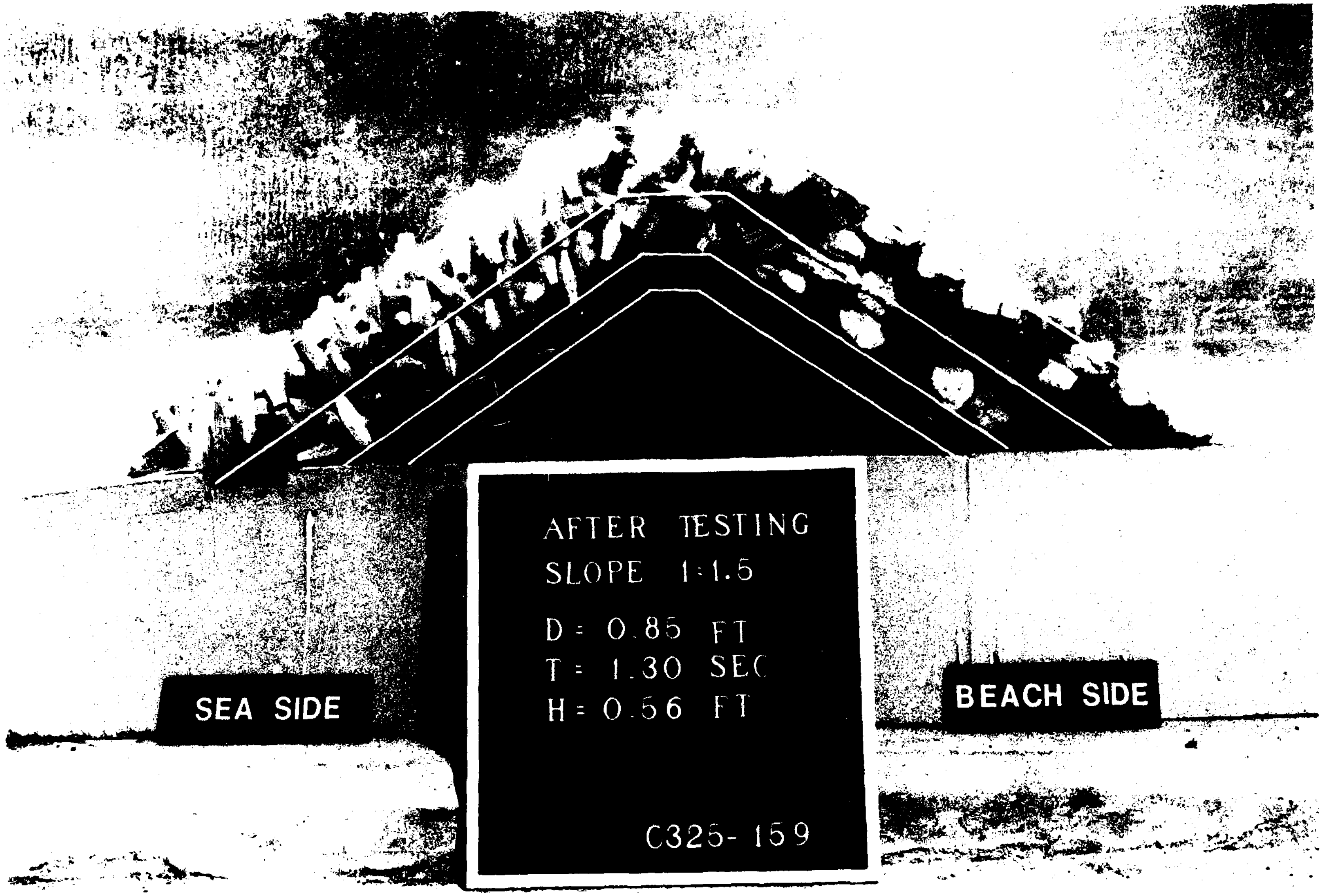

Photo 6. End view after attack of 1.30-sec, 0.56-ft waves; d = 0.85 ft; W_a = 0.442 lb; 1V-on-1.5H-structure slope

Photo 7. Sea-side view after attack of 2.32-sec, 0.58-ft waves; d = 0.60 ft; W_a = 0.589 lb; 1V-on-1.5H-structure slope

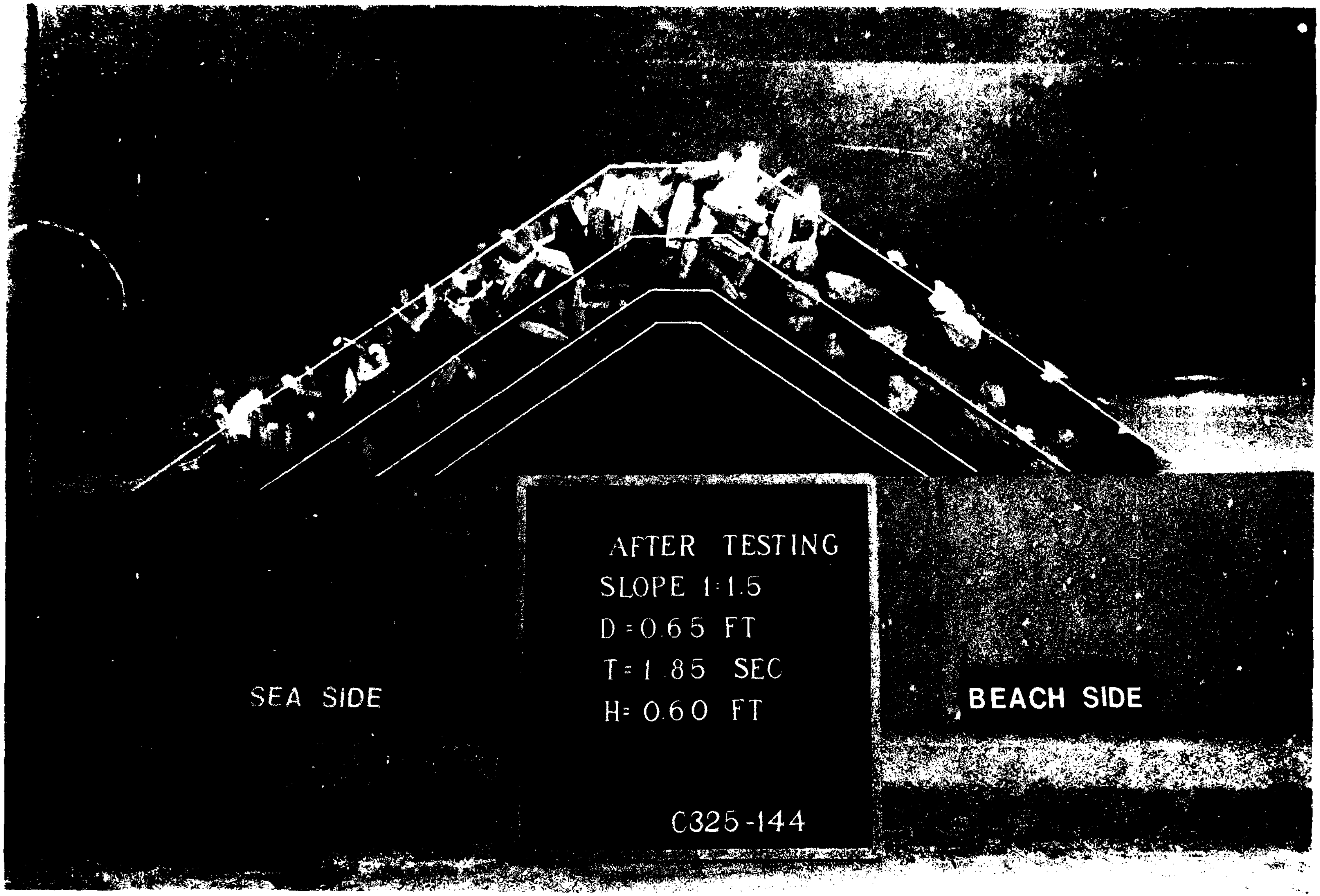

Photo 8. End view after attack of 1.85-sec, 0.60-ft waves; d = 0.65 ft; W_a = 0.589 lb; 1V-on-1.5H-structure slope

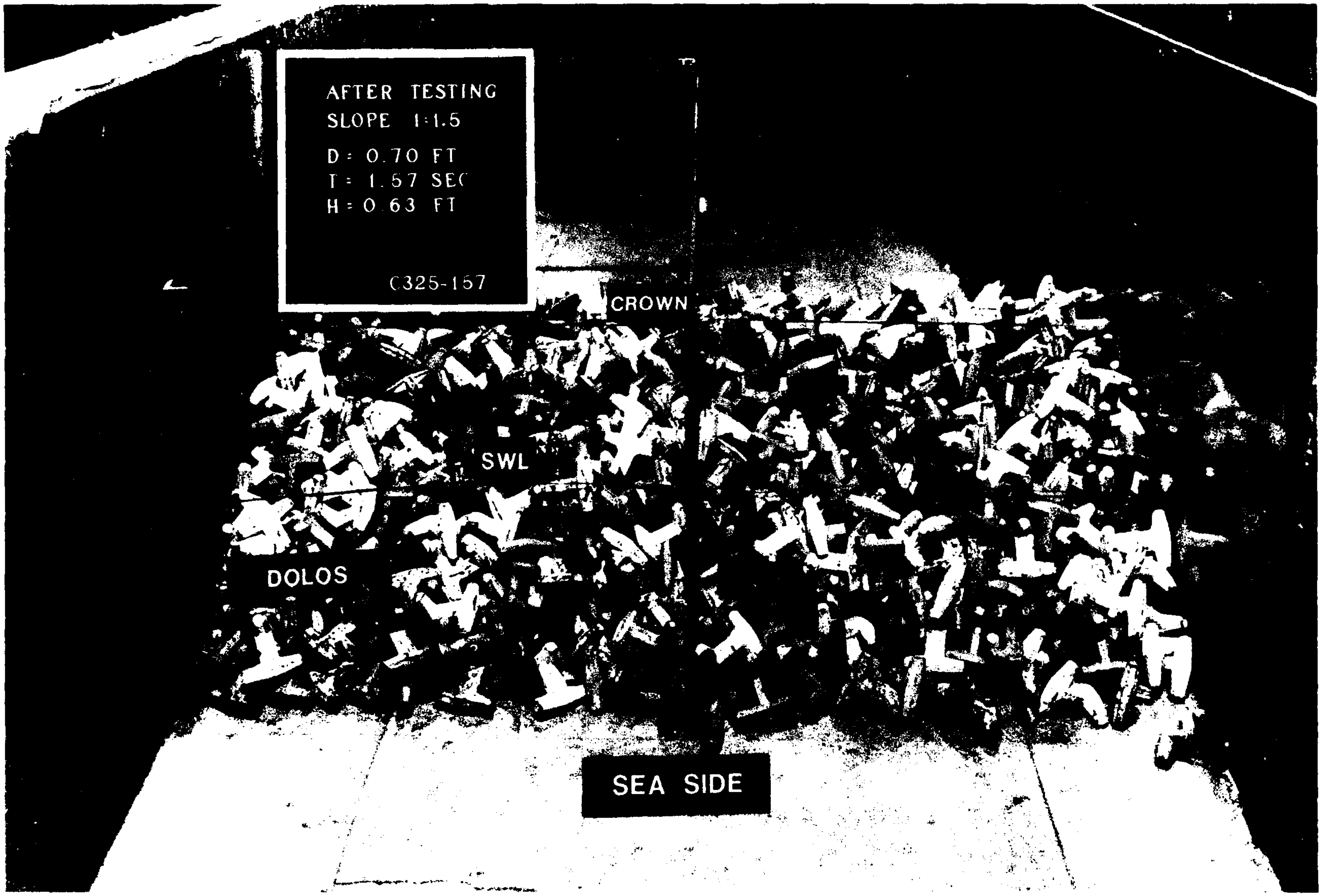

Photo 9. Sea-side view after attack of 1.57-sec, 0.63-ft waves; d = 0.70 ft; W_a = 0.589 lb; 1V-on-1.5H-structure slope

Photo 10. Sea-side view after attack of 1.47-sec, 0.63-ft waves; d = 0.85 ft; W_a = 0.589 lb; 1V-on-1.5H-structure slope

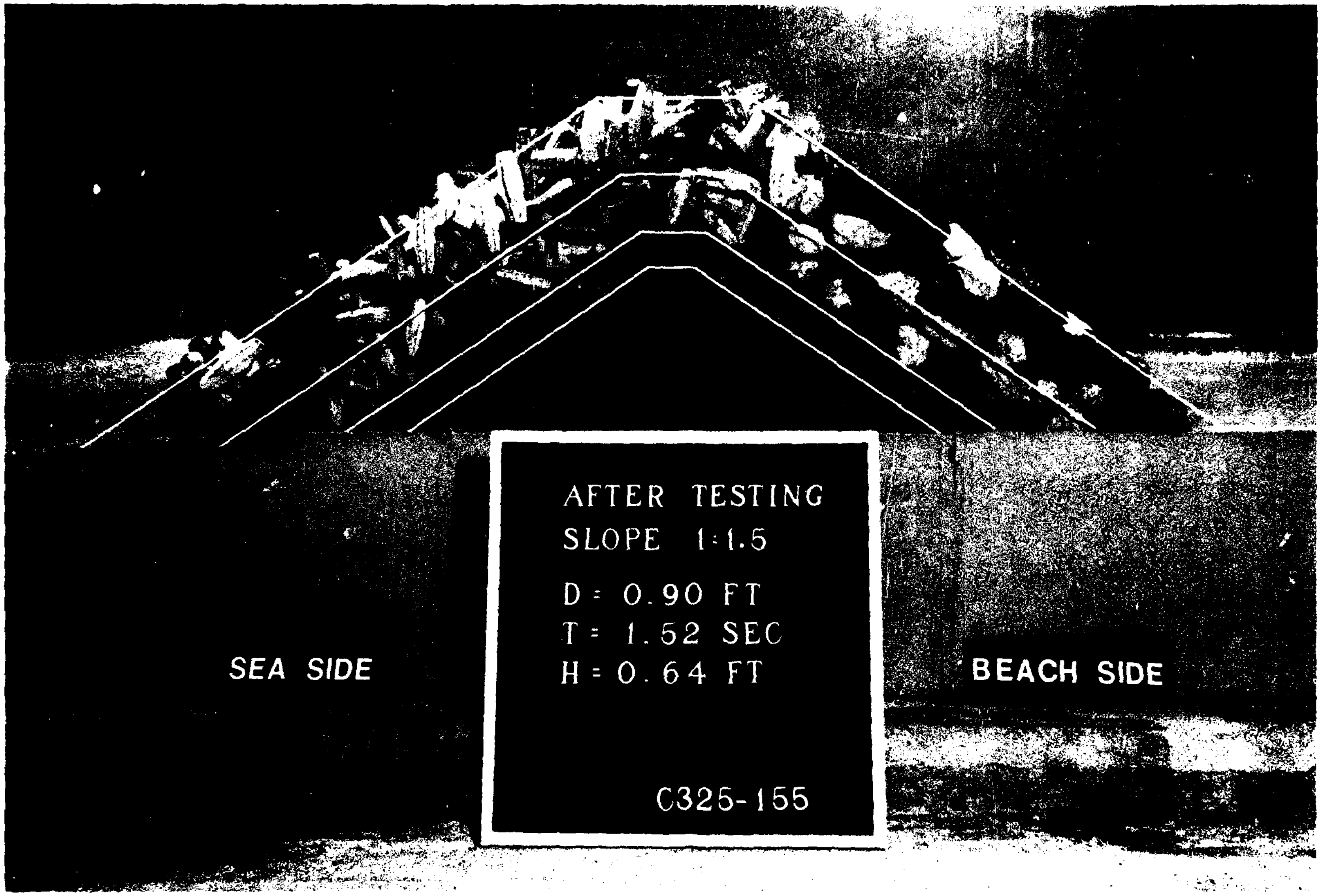

Photo 11. End view after attack of 1.52-sec, 0.64-ft waves; d = 0.90 ft; W_a = 0.589 lb; 1V-on-1.5H-structure slope

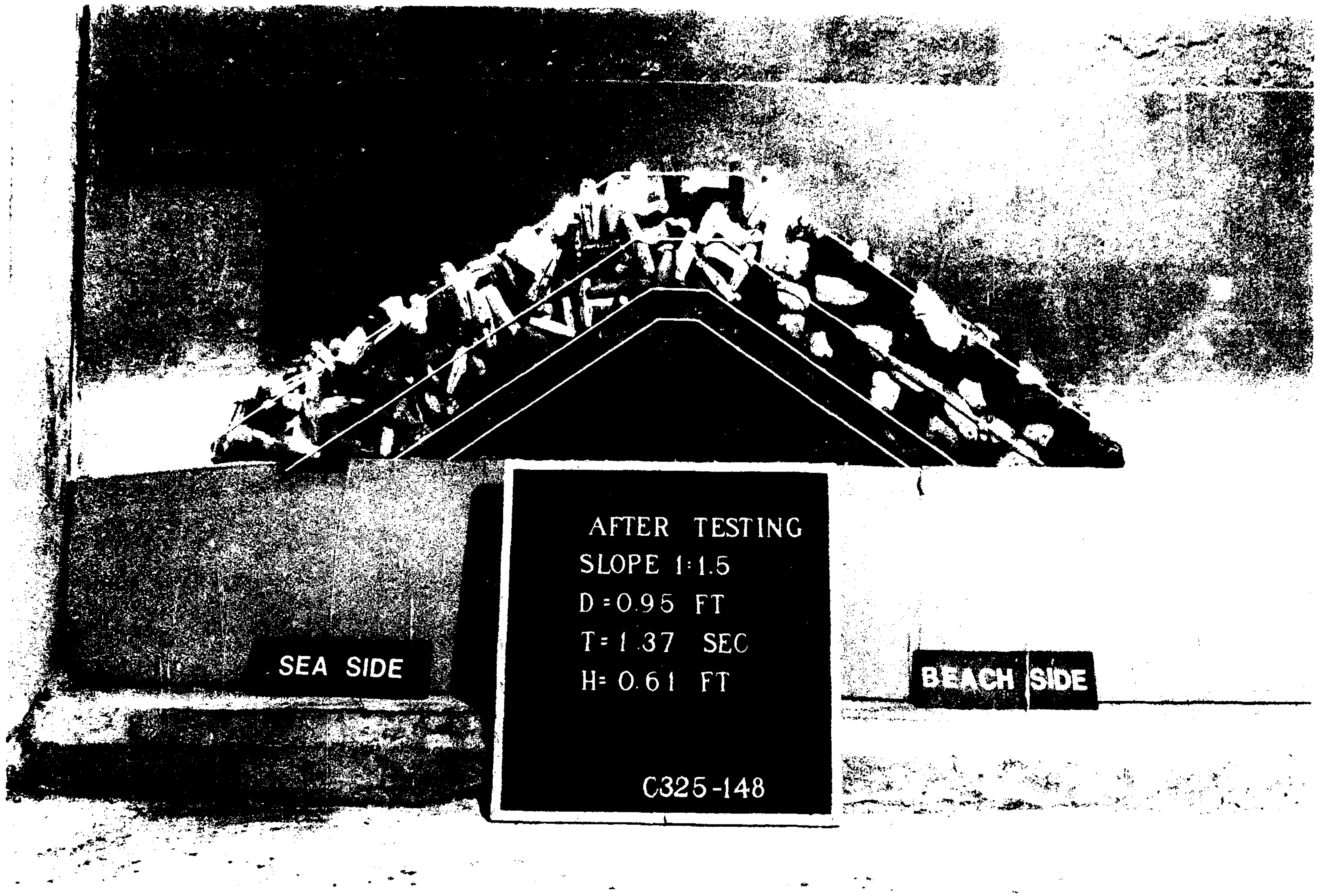

Photo 12. End view after attack of 1.37-sec, 0.61-ft waves; $d = 0.95$ ft; $W_a = 0.589$ lb; 1V-on-1.5H-structure slope

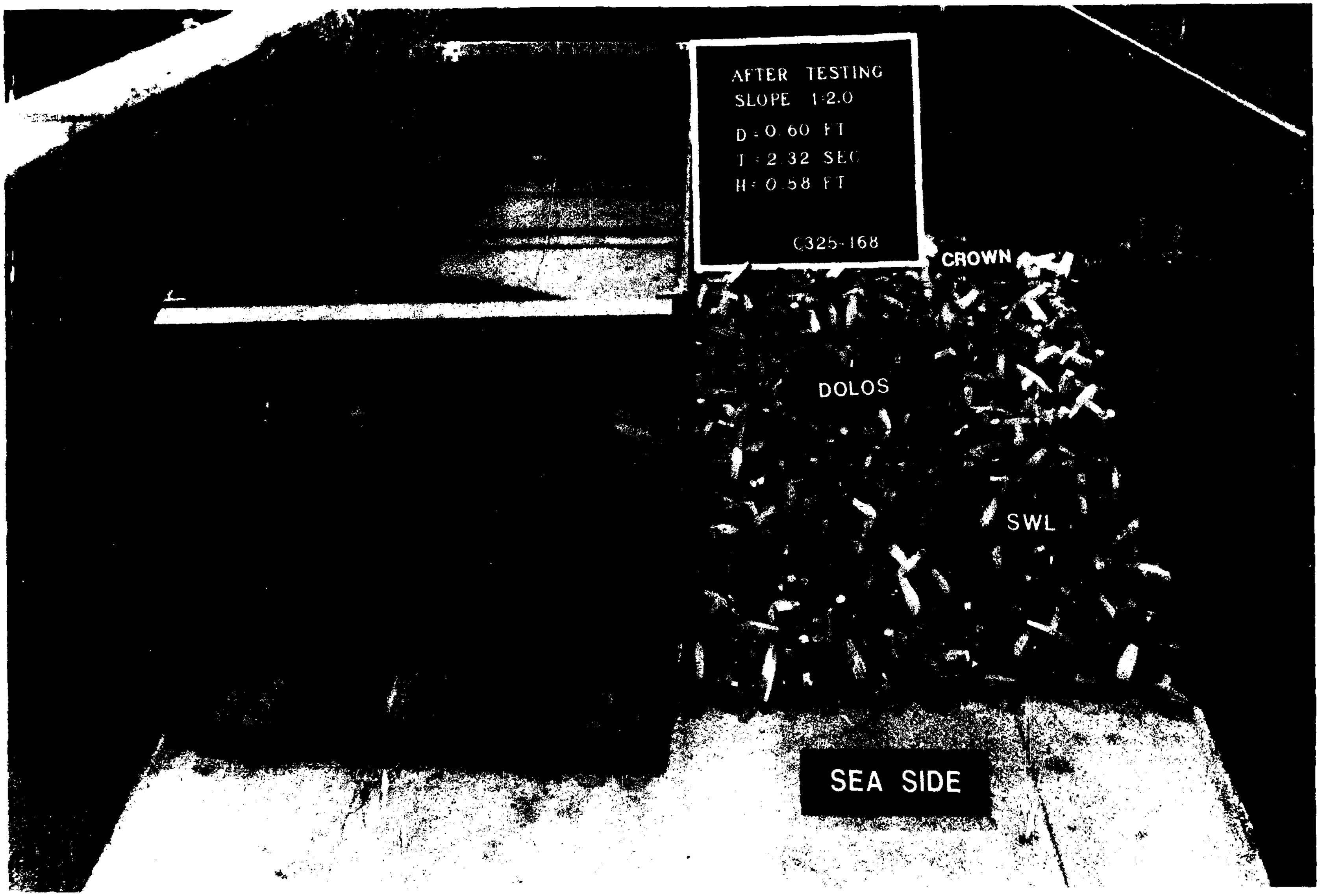

Photo 13. Sea-side view after attack of 2.32-sec, 0.58-ft waves; d = 0.60 ft; W_a = 0.442 lb; 1V-on-2H-structure slope

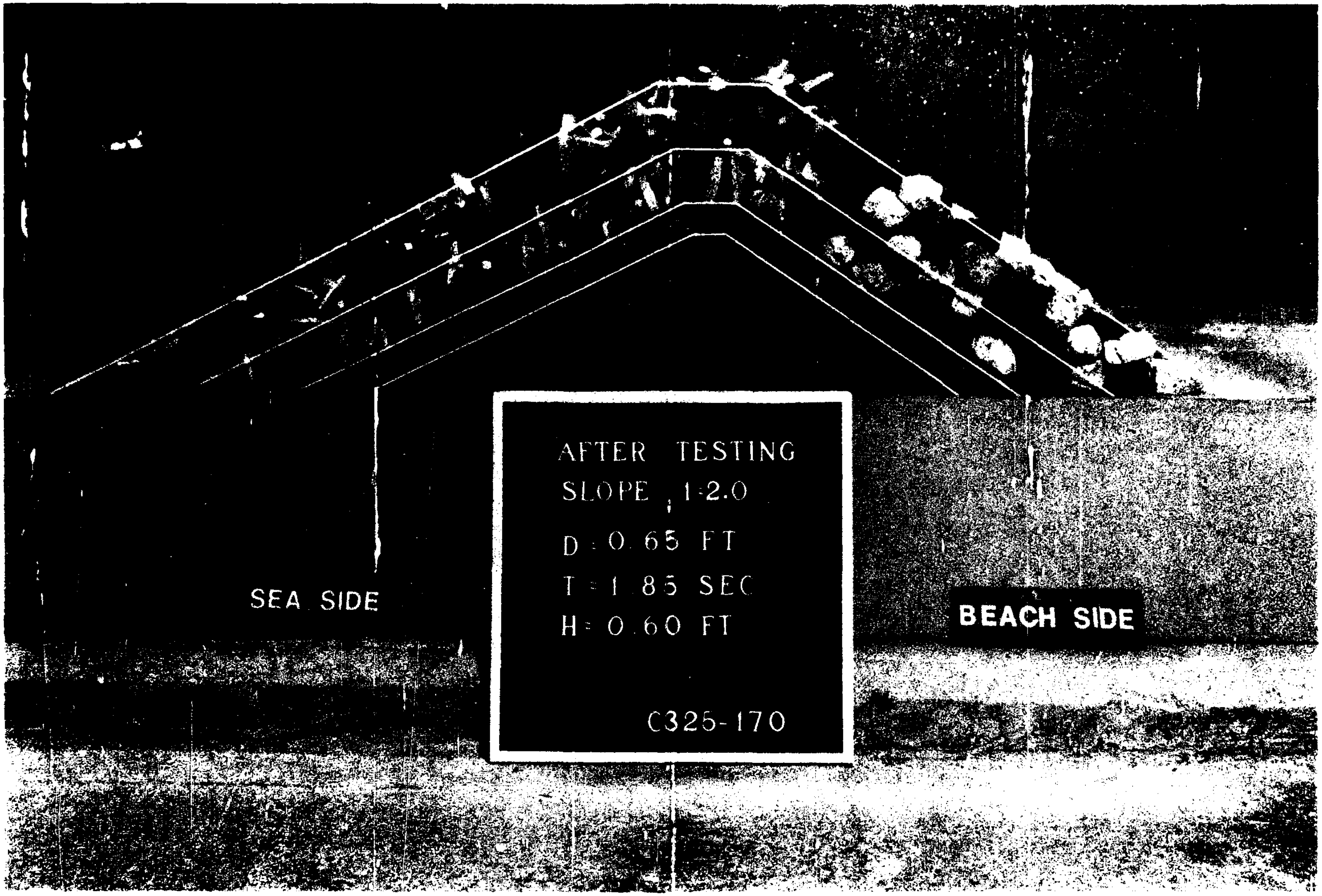

Photo 14. End view after attack of 1.85-sec, 0.60-ft waves; d = 0.65 ft; W_a = 0.442 lb; 1V-on-2H-structure slope

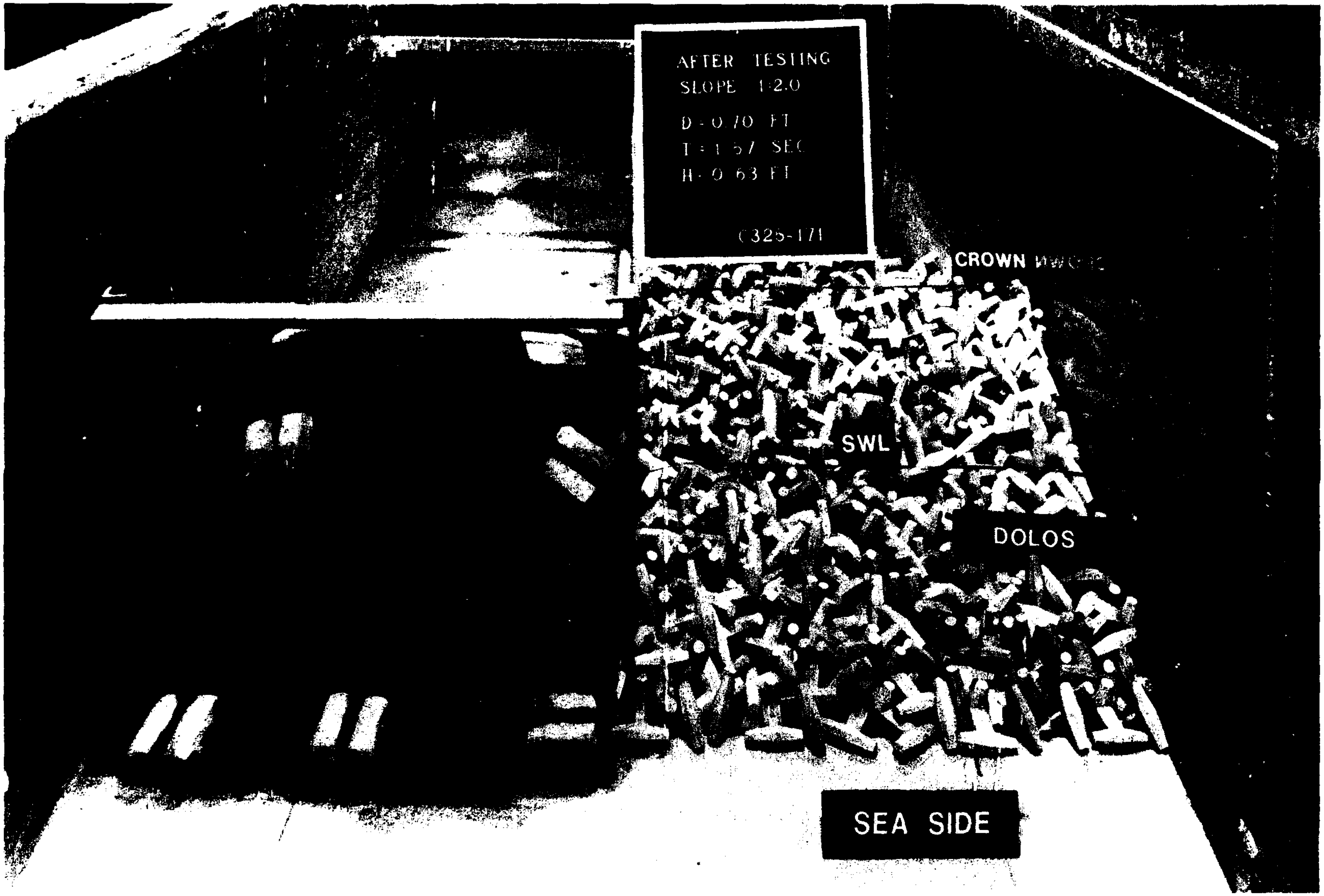

Photo 15. Sea-side view after attack of 1.57-sec, 0.63-ft waves; d = 0.70 ft; W_a = 0.442 lb; 1V-on-2H-structure slope

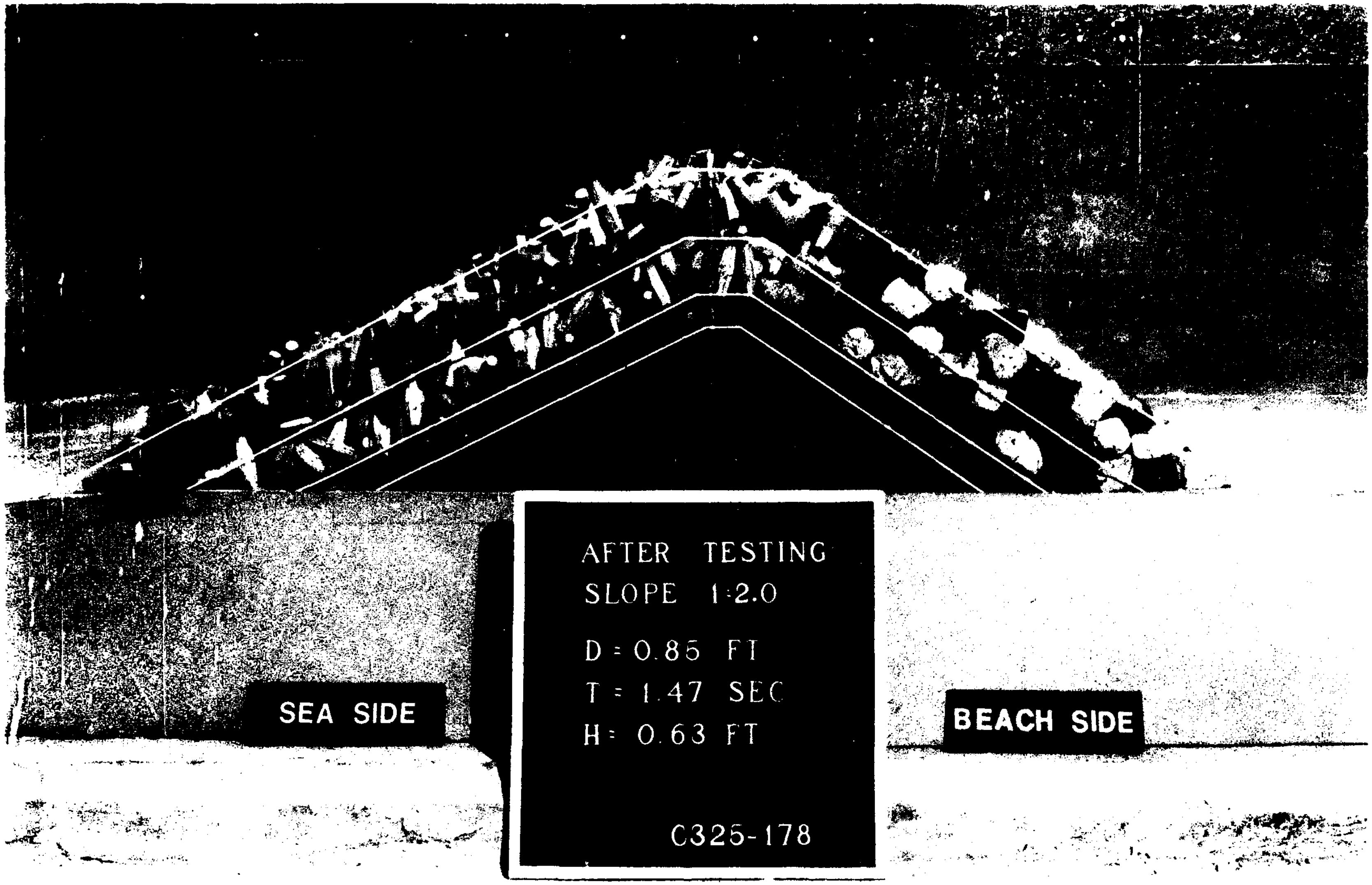

Photo 16. End view after attack of 1.47-sec, 0.63-ft waves; d = 0.85 ft; W_a = 0.442 lb; 1V-on-2H-structure slope

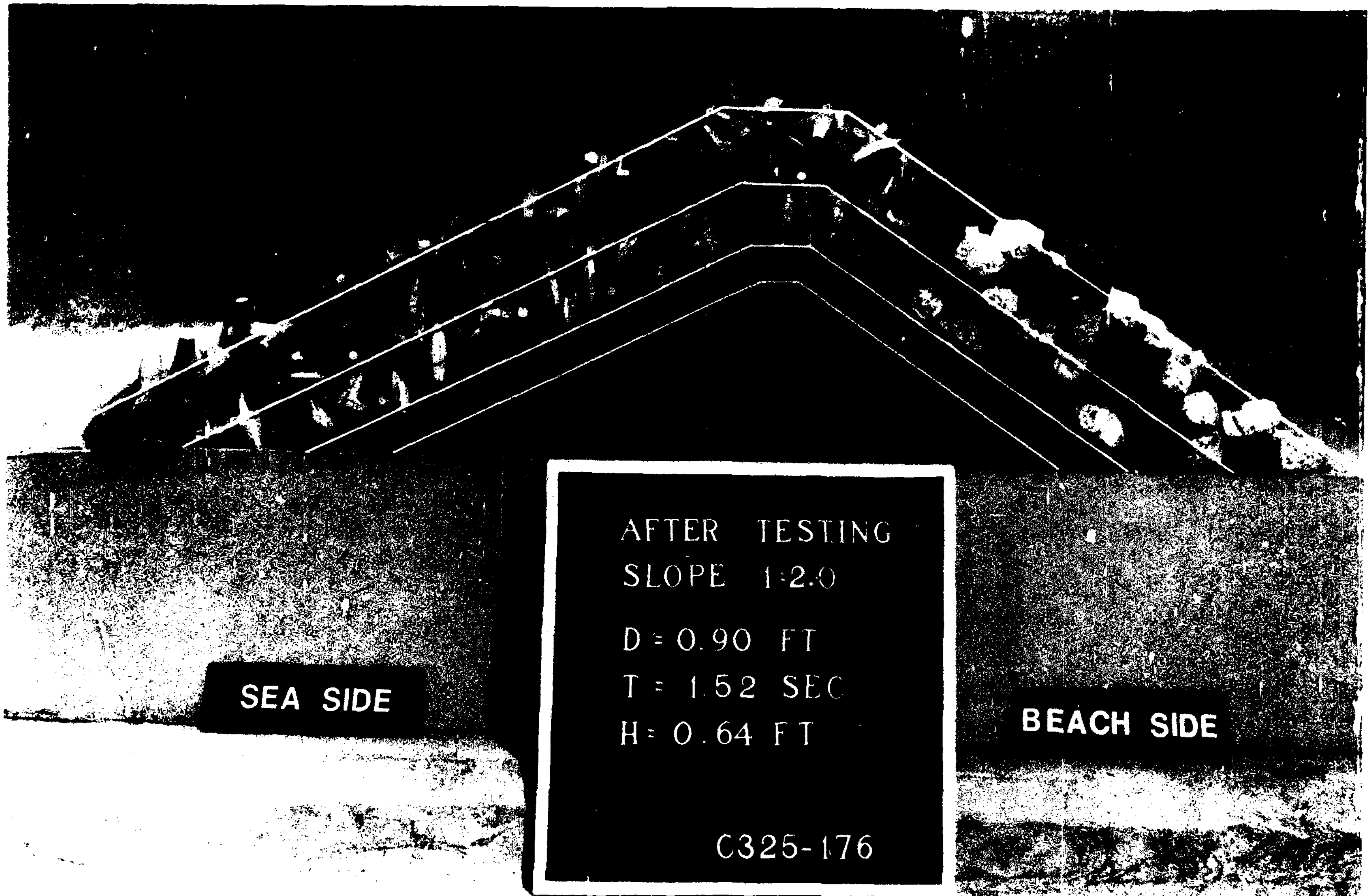

Photo 17. End view after attack of 1.52-sec, 0.64-ft waves; d = 0.90 ft; W_a = 0.442 lb; 1V-on-2H-structure slope

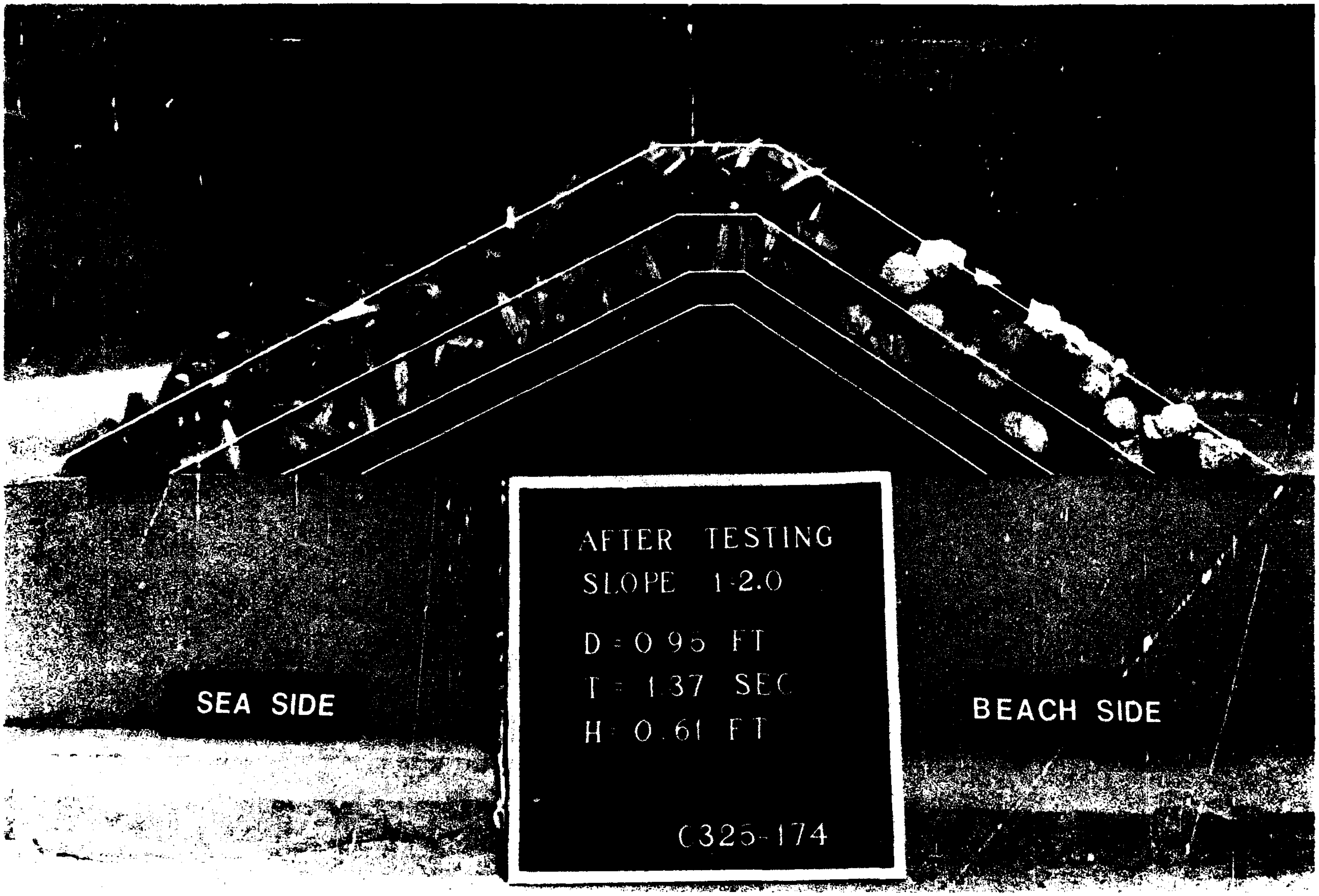

Photo 18. End view after attack of 1.37-sec, 0.61-ft waves; d = 0.95 ft; W_a = 0.442 lb; 1V-on-2H-structure slope

Photo 19. Sea-side view after attack of 1.99-sec, 0.70-ft waves; d = 0.75 ft; W_a = 0.589 lb; 1V-on-2H-structure slope

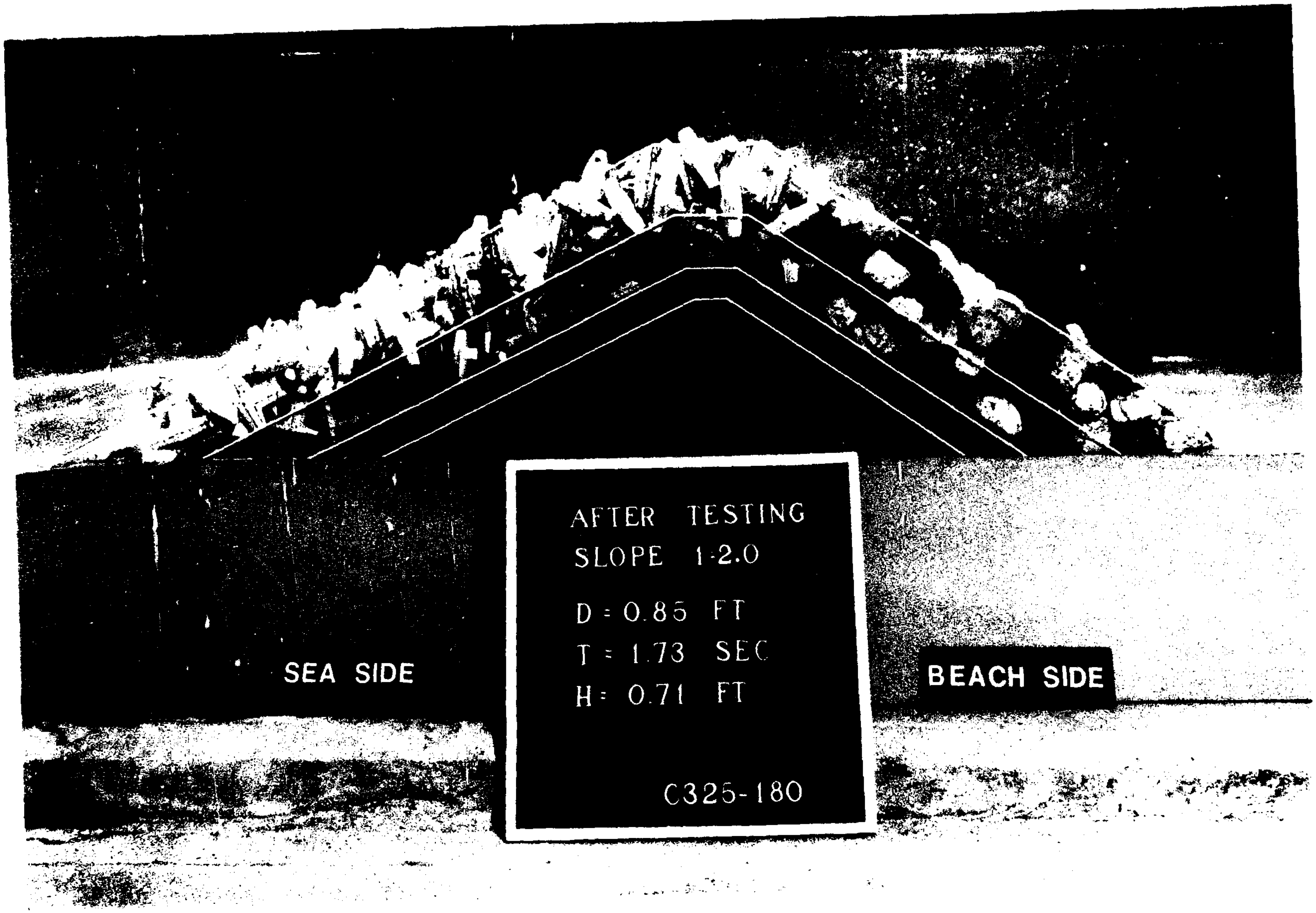

Photo 20. End view after attack of 1.73-sec, 0.71-ft waves; d = 0.85 ft; W_a = 0.589 lb; 1V-on-2H-structure slope

APPENDIX A: NOTATION

A	Surface area, ft^2
c	Coefficient
d	Water depth, ft
d/L	Relative depth
g	Acceleration due to gravity, ft/sec^2
H	Wave height, ft
H/d	Relative wave height
k_Δ	Shape coefficient
K_D	Stability coefficient
ℓ_a	Characteristic length of armor unit, ft
L	Length, wavelength, ft
n	Number of layers of armor units
N	Number of armor units
P	Porosity of breakwater material, percent
R_N	Reynolds stability number = $g^{1/2}H^{1/2}\ell_a/\nu$
T	Wave period, sec; time
S_a	Specific gravity of armor unit
$\forall$	Volume, ft^3
W	Weight, lb
α	Angle of breakwater slope, measured from horizontal, deg
$\cot \alpha$	Reciprocal of breakwater slope
γ	Specific weight, pcf
γ_a	Specific weight of an armor unit, pcf
Δ	Shape or armor unit of underlayer material
ν	Kinematic viscosity

<u>Subscripts</u>

a	Refers to armor unit
s	Refers to stability
w	Refers to water in which the structure is located

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